GEOTECHNICAL ENGINEERING INVESTIGATION
LINDEN LANE BRIDGE
MARIN COUNTY, CALIFORNIA

For

NOLTE ASSOCIATES, INC.
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August 2006 Job No. 205152.LDN

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GEOTECHNICAL ENGINEERING INVESTIGATION
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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed new bridge in the City of San Rafael, California. Our work was performed generally in accordance with the scope of work as per our agreement. The general location of the site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. These recommendations should not be used for direct bidding purposes.

PROPOSED CONSTRUCTION

The proposed Linden Lane Bridge is part of the Puerto Suello Multi-Use Path Improvement Project. Based on the plan set provided by the designer, the proposed bridge will be located approximately 6 meters west of Linden Lane Undercrossing (Br. No. 27-0034), crossing over Linden Lane in the City of San Rafael, California.

The proposed bridge will be a single-span cast-in-place concrete bridge of approximately 22.7 m in length and approximately 3.7 m in width. It is planned to use 600 mm (2 feet) diameter Cast-In-Drilled-Hole (CIDH) concrete piles for abutment foundation support.

PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties, and to provide recommendations for foundation support of the proposed bridge structure.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site, engineering analysis of the as-built boring data, and preparation of this report. The basis for this investigation is the General Plan and Foundation Plan provided to us by Nolte Associates, Inc. We have presented the log of test borings in Appendix A.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

Our recommendations in this report are based on the above information. Any major deviation should be reported to this office for consideration.

SITE CONDITIONS

The project site is approximately 6 meters west of the intersection of HWY 101 and Linden Lane. The proposed bridge alignment will be located in-between the existing Union Pacific Railroad structure and Linden Lane Undercrossing (Br. No. 27-0034) at HWY 101, crossing over Linden Lane in the City of San Rafael, California. Based on the profile plans provided by the clients, the lowest part of the existing Linden Lane is approximately at Elev. 15 m. The area is generally residential.

FIELD EXPLORATION

No new exploration was performed for the proposed bridge. Our analysis was based on two as-built borings drilled for Linden Lane Undercrossing (Widen) in 1983. The exploratory borings were explored to a maximum depth of 15.5 m (51 feet) below the existing ground surface. The locations and description of the materials are shown on Site Plan, Plate No. 2 and Appendix A of the report.

According to the as-built information, the test borings were advanced using rotary wash drilling method. Selected samples were obtained from Standard Penetration Test samplers (35 mm I.D.) at various depths. The SPT samplers were driven under the impact of a 63.5 kg hammer having a free fall of 76 cm. The blow counts are presented on the "Log of Test Borings". Laboratory data of these two borings are not available. The bore logs presented in Appendix A were reconstructed by Caltrans based on "Vintage Log of Test Borings" from earlier foundation investigations performed by Structures Design during 1983 and 1984.

SUBSURFACE CONDITIONS

Boring B-3, located at the proposed Abutment 1 (south), encountered stiff to hard silty clay to Elev. 8.5 m, overlying medium dense sand to very dense gravel to Elev. 6.1 m. Rock formation, such as Sandstone, Greenstone and Basalt, was encountered below Elev. 6.1 m to Elev. 0, the maximum depth drilled. Boring B-2, located at the proposed Abutment 2 (north), encountered dense to very dense sand to Elev. 13.1 m, overlying stiff sandy lean clay to Elev. 7.0 m. Conglomerate and Serpentinized Basalt, were encountered below previous depth to Elev. 4.3 m, the maximum depth drilled.

The groundwater level was not measured due to the rotary wash method of drilling. However, groundwater was encountered at Elev. 11.7 m in Boring B-4, located approximately 23 m south of B-3, and was also encountered at Elev. 22.4 m in Boring B-9, located approximately 165 m north of B-2 during exploration. Based on the head difference, it is reasonable to assume that the groundwater is flowing toward south within this section.

For design purpose, it is prudent to assume the groundwater at approximately Elev. 16 m at the project site. The groundwater level may vary with the passage of time due to seasonal groundwater fluctuation, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of investigation.

GEOLOGY

General geologic features pertaining to the site were evaluated by reference to the Geologic Map & Map Database of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties California, by M. C. Blake Jr. (USGS, MF-2337, Version 1). Based on the publication, the site is generally underlain by Mélange, a tectonic mixture of variably sheared shale and sandstone. A geologic map of the project vicinity is shown on Plate 3.

EARTHQUAKE CONSIDERATIONS

Seismic Sources

The project site is located in a seismically active part of northern California. Many faults exist in the San Francisco Bay Area that are capable of producing earthquakes that may cause strong ground shaking at the site. Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (1996) are summarized below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

Fault	Distance from Site (km)	Maximum Credible Earthquake Magnitude
San Andreas	~15	8
Hayward	~12	7½

Active faults in the vicinity include the San Andreas Fault and Hayward Fault. A major earthquake on these faults can produce strong ground shaking at the site. A Fault Map of the general project vicinity is shown on Plate 4. Based on the seismic hazard map prepared by Mualchin (1996) and attenuation relationship proposed by Sadigh, et al (1997), a Peak Bedrock Acceleration (PBA) of 0.4 g is anticipated at the site.

Seismic Hazards/Liquefaction Impact

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the proposed bridge structures, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered moderate to high.

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged, cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

Based on the boring data, sand and gravel layers were encountered in both of the borings. Per discussion with the reviewer, the medium dense sand layer encountered at the south abutment (Abut 1) may liquefy if the groundwater elevation is lower than our assumption. Based on our analysis, the medium dense sand layer will be subject to liquefaction when the groundwater level is lower than

Elev. 10 m, and the post-liquefaction settlement is anticipated to be on the order of 13 mm (0.5"). To be conservative, liquefaction was considered for pile capacity estimation.

FINDINGS AND RECOMMENDATIONS

General

Based on the findings of our investigation, it is our opinion that the site is feasible for the planned construction. The recommendations presented in this report are incorporated into the final design and construction.

This report was prepared specifically for the proposed bridge structure according to the plans provided to us. Normal construction procedures were assumed throughout our analysis and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials encountered on the site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

Foundations

Based on the General Plan and Foundation Plan provided by the designer, CIDH concrete piles are considered for the proposed bridge. According to the boring data, the subsurface conditions consist of predominantly cohesive material with sand lenses at Abutment 1, and dense sand over stiff clay at Abutment 2. Groundwater is expected for pile construction. Per discussion with the designer, 600 mm (24-inch) diameter CIDH concrete piles are being considered for the abutments support. At Abutment 1, a 900 mm diameter storm drainage pipe will be installed between the CIDH piles. The invert of the pipe is at approximately Elev. 12.7 m, and the distance between the proposed pipe and the CIDH piles is about 250 mm on both side. Therefore, it is reasonable to neglect the vertical capacity within the 1.5 m zone below the footing for the backfilling material.

Per discussion with the reviewer, the medium dense encountered at Abutment 1 may liquefy with groundwater level lower then Elev. 10 m. Therefore, we have neglected the vertical capacity in the upper zone of the pile. Based on our analysis, the post-liquefaction settlement is anticipated to be on the order of 13 mm (0.5"). Down drag force should be minimal for such small movement. Based on our analyses, CIDH concrete piles are appropriate for the foundation of the structure. The following table summarizes the pile foundation recommendations.

PILE DATA TABLE

Location	Pile Type	Design Loading (Service)	Nominal Resistance		Cut-off Elev. (m)	Design Tip Elev. (m)	Specified Tip Elev. (m)
			Compression	Tension			
Abut 1	600mm CIDH	516 kN	1032 kN	--	14.25	2.0 (1); 6.6 (2)	2.0
Abut 2	600mm CIDH	516 kN	1032 kN	--	16.17	5.4 (1); 6.9 (2)	5.4

*Design Tip Elevation is controlled by the following demands: (1) Compression; (2) Lateral Loads

Based on the soil and groundwater conditions, the use of temporary steel casing shall be expected to maintain the integrity of the piles. Caltrans standard specification for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH concrete piles. Pile excavations should not be allowed to stand open overnight, and excavations should be poured as soon as possible. The bottom of the pier excavations should be free of debris and loose materials and properly cleaned. Access tubes should be provided to allow for construction quality control (gamma-gamma logging).

Due to presence of sandy material and groundwater, ravelling or caving is expected which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. It is prudent to make the contractor aware of these conditions so that he takes appropriate steps to comply with the standards and maintain the integrity of the piles. All piles excavations should be observed by the geotechnical engineer or regulatory agency prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

Lateral Pile Capacity

Lateral load analyses were performed for the planned 600 mm (24-inch) diameter CIDH piles at Abutment 1. Between the pile cap and piles, it will be designed as a "fixed-head" connection. The analyses of the abutment piles considered group efficiency, and a factor of 0.6 (60% of the original soil p-y relationship) is recommended for pile spacing of 3D. As mention in the previous paragraph, a 900 mm diameter storm drainage pipe will be installed between the CIDH piles. Therefore, we have also neglected the lateral resistance within 1.5 m below the footing. Liquefaction was considered for the lateral pile capacity analysis. Plots of deflection, moment, shear and soil reaction along the pile length are attached in Appendix C of the report.

Seismic Design Criteria

Based on the seismic hazard map prepared by Mualchin (Caltrans, 1996), the governing faults for the structure consist of the following faults: (1) the San Andreas Fault (a strike-slip fault, Mw=8.0), located at about 15 km from the site with an anticipated Peak Bedrock Acceleration of 0.4g; (2) the Hayward Fault (a strike-slip fault, Mw=7.5), located at about 12 km from the site with an anticipated Peak Bedrock Acceleration of 0.4g.

The recommended curve is based on Caltrans Seismic Design Criteria (Version 1.3, February 2004). A fault map and the ARS Design Curve are attached with this letter. The seismic design criteria are as follows:

1. Soil Profile D.
2. ARS Design Curve – an envelope of the following two curves:
   A) Modified Figure B.9 (SDC 1.3), Mw = 8.0, PBA = 0.4g with 20 % increase of Sa for structual periods ≥ 1 second, no change of Sa for structural periods <0.5 seconds, linear interpolation of Sa between 0.5 and 1 seconds to account for near-fault effect (for the San Andreas Fault which governs long period range).

B) Modified Figure B.8 (SDC 1.3), Mw = 7.5, PBA = 0.4g with 20 % increase of Sa for structual periods ≥ 1 second, no change of Sa for structural periods <0.5 seconds, linear interpolation of Sa between 0.5 and 1 seconds to account for near-fault effect (for the San Andreas Fault which governs long period range).

Corrosion

No corrosion test result is available from the as-built boring data. Two corrosion tests were performed on selected samples obtained from B-2 and B-5, drilled by Parikh consultants, Inc. in February 2006. B-2 and B-5 are located at the intersection of Lincoln Avenue and the proposed trail, approximately 0.9 km north of the project site. The corrosion investigations performed are in general accordance with the provisions of California Test Method 643. A summary of the corrosion test data is presented below.

Boring	Depth (m)	PH	Resistivity (ohms-cm)	Sulfate (ppm)	Chloride (ppm)
B-2	3.4	6.81	2570	61.0	5.7
B-5	1.8	6.94	2600	100.6	8.9

Based on the data, the site is considered non-corrosive per Caltrans corrosion design guideline, and standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructures. The minimum cement factor and cover thickness should be per Caltrans Bridge Design Specifications (Section 8.22).

Plan Review

We recommend that final plans for foundations be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of foundation excavations, and pile installations should be carried out by the regulating agencies. If the subsurface conditions different from those forming the basis of our recommendations is encountered this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

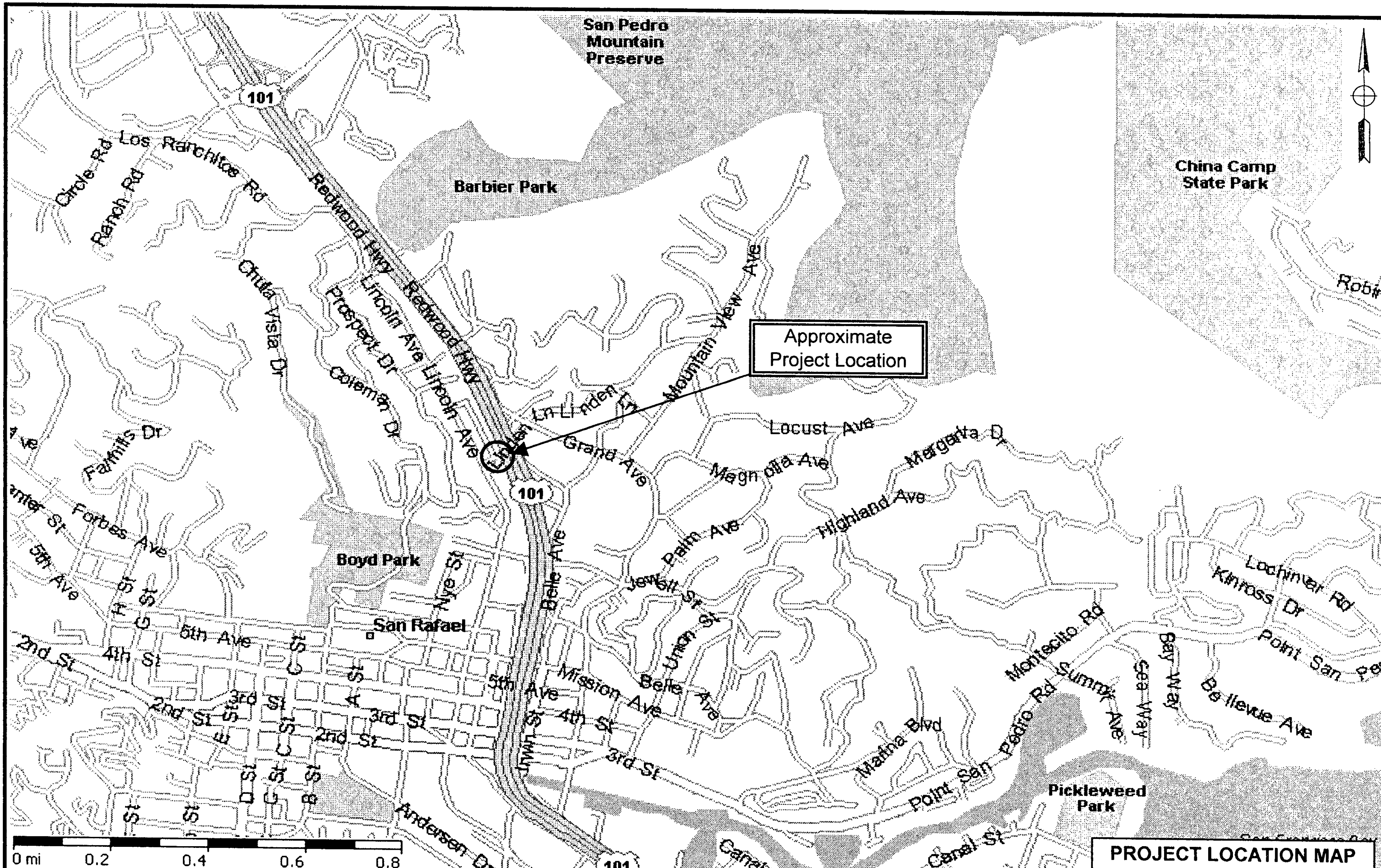
Respectfully submitted,
PARIKH CONSULTANTS, INC.

Frank Y. Wang, P.E. 67731
Project Engineer

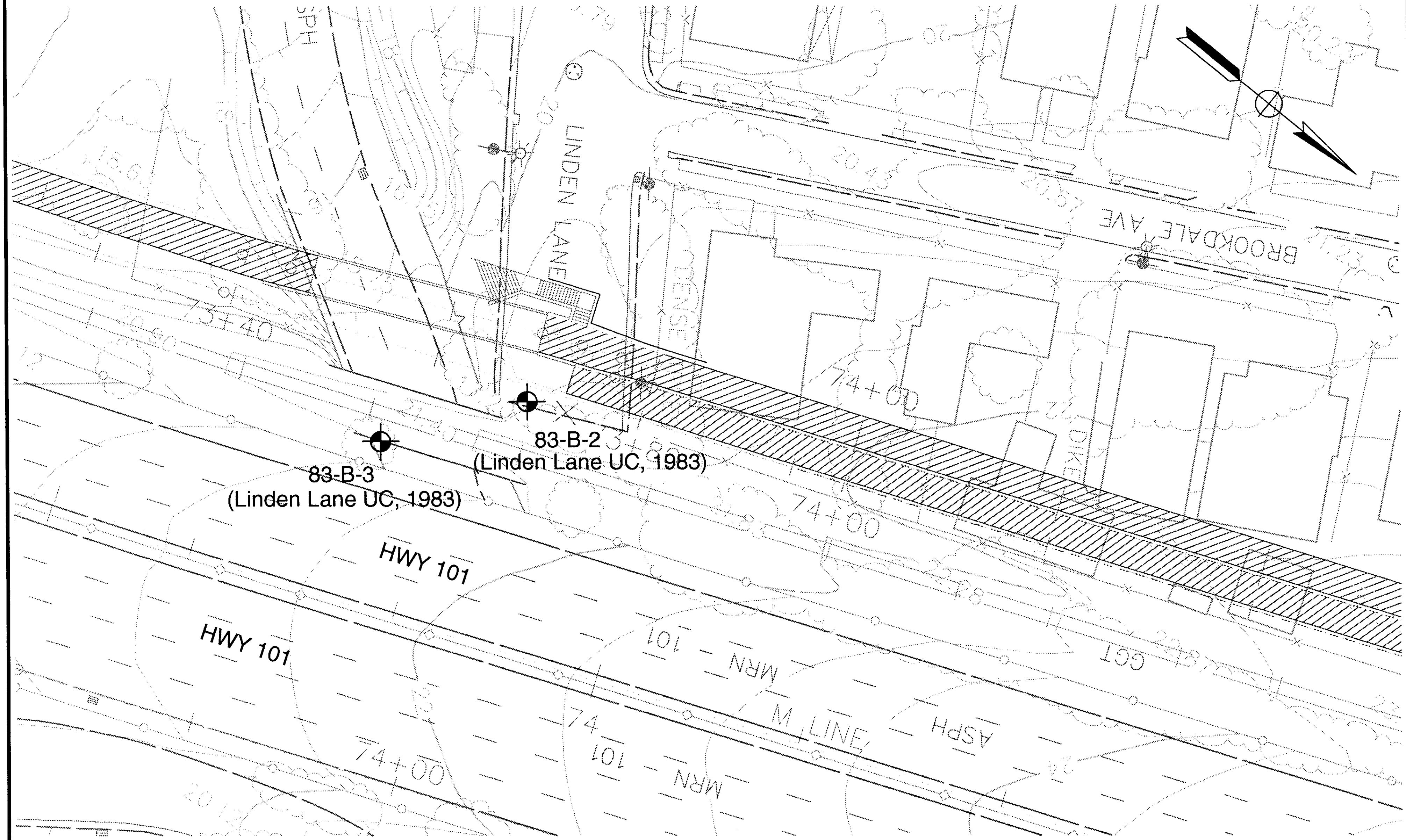
REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-07
GEOTECHNICAL
STATE OF CALIFORNIA

Gary Parikh, G.E. 666
Project Manager

Report (Linden Br) {Ongoing Projects\205152.10\}



PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	LINDEN LANE BRIDGE SAN RAFAEL, CALIFORNIA
JOB NO.: 205152.10	PLATE NO.: 1



LEGEND

Approx. As-Built Boring Location (by others)

SCALE 1:500

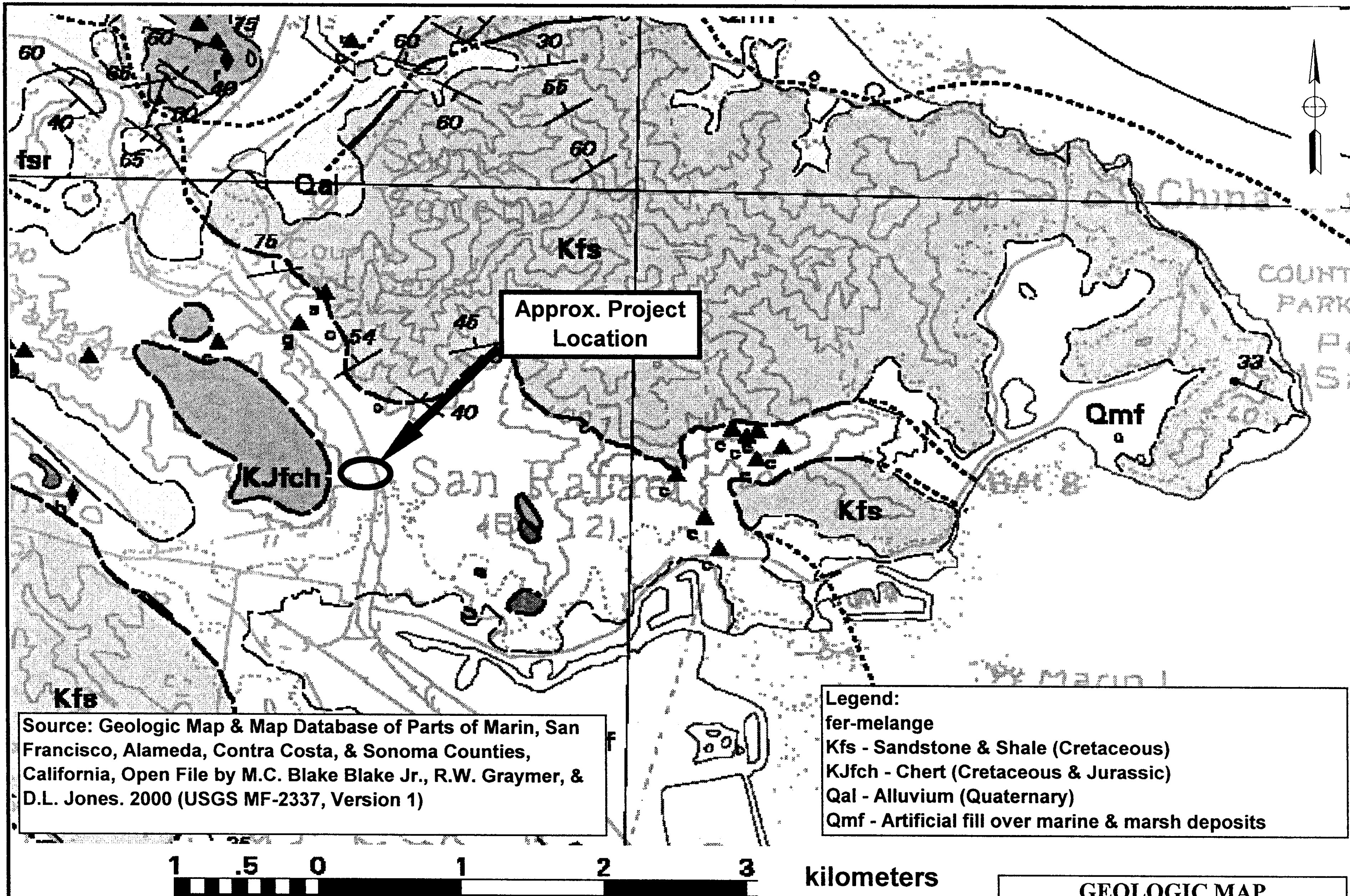
Note:
All units are in meters unless otherwise specified.
Reference Map was provided by NOLTE ASSOCIATES.

SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

LINDEN LANE BRIDGE
MARIN COUNTY, CALIFORNIA

JOB NO.:205152.LDN	PLATE NO. 2



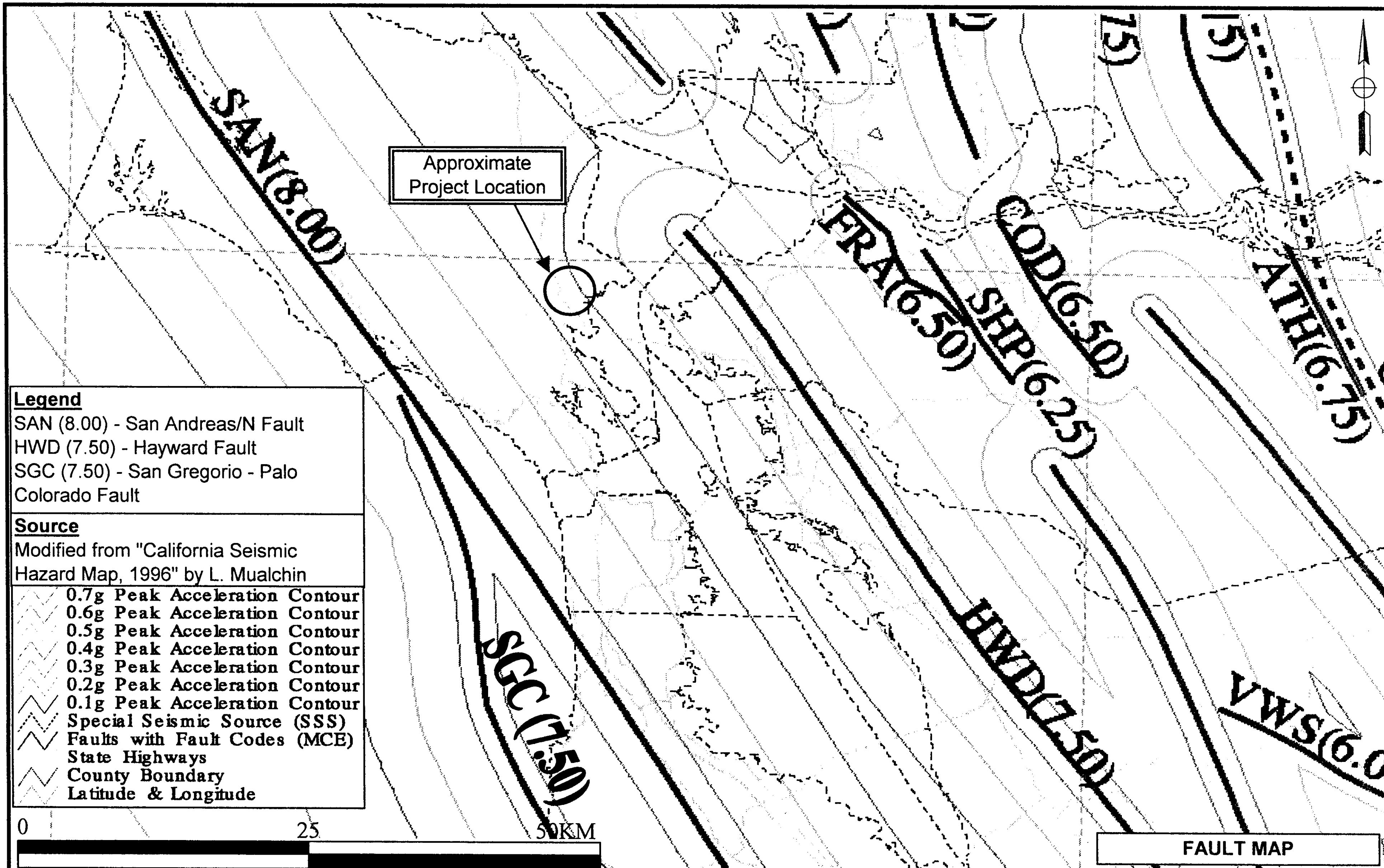
GEOLOGIC MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

LINDEN LANE BRIDGE
SAN RAFAEL, CALIFORNIA

JOB NO.: 205152.10

PLATE NO.: 3



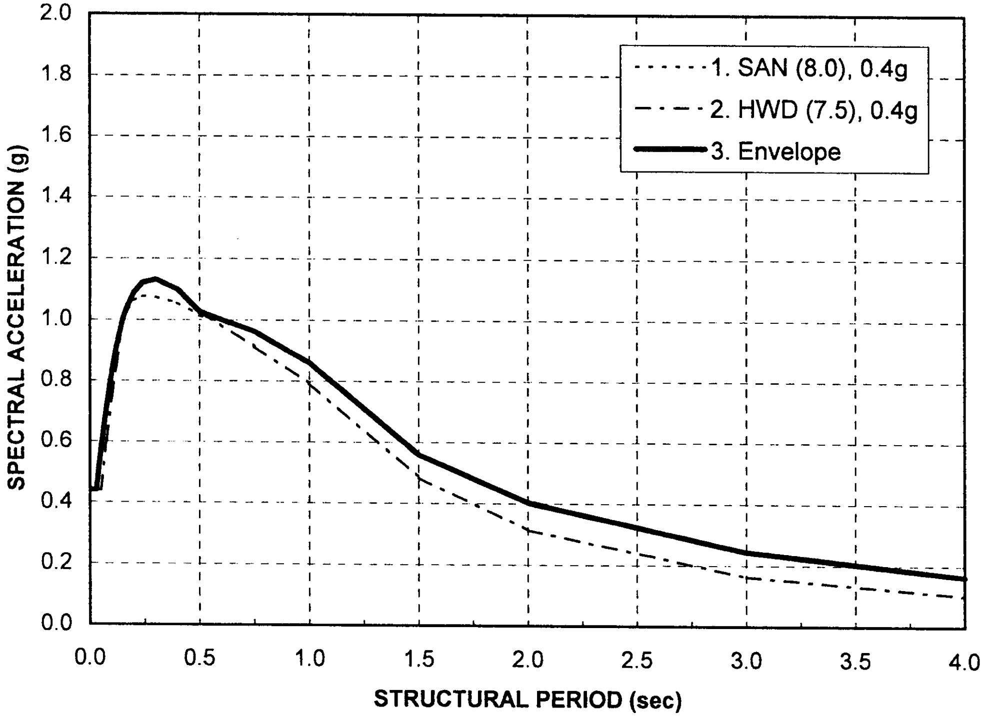
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

LINDEN LANE BRIDGE
SAN RAFAEL, CALIFORNIA

JOB NO.: 205152.10

PLATE NO.: 4

ARS DESIGN CURVE
LINDEN LANE BRIDGE
MARIN COUNTY, CALIFORNIA



Spectral Data

Period (sec)	SAN (8.00) Spectral Accel. (g)	HWD (7.50) Spectral Accel. (g)
0.010	0.440	0.440
0.020	0.440	0.440
0.030	0.440	0.440
0.050	0.576	0.440
0.075	0.713	0.605
0.100	0.830	0.747
0.120	0.906	0.845
0.150	0.994	1.003
0.170	1.030	1.040
0.200	1.062	1.087
0.240	1.076	1.120
0.300	1.074	1.129
0.400	1.054	1.098
0.500	1.015	1.026
0.750	0.961	0.913
1.000	0.860	0.792
1.500	0.560	0.485
2.000	0.403	0.314
3.000	0.243	0.164
4.000	0.164	0.101

1. Caltrans SDC (v 1.3, February 2004), Figure B.9,
Governing Fault: San Andreas Fault
(Mw = 8.0, Soil Profile Type D, PBA = 0.4 g)
with the following modifications:
(1) No change of Sa for structural periods < 0.5 sec
(2) 20% increase of Sa for structural periods $\geq$ 1 sec
(3) Linear interpolation for structural periods between 0.5 and 1 sec

2. Caltrans SDC (v 1.3, February 2004), Figure B.8,
Governing Fault: Hayward Fault
(Mw = 7.50, Soil Profile Type D, PBA = 0.4 g)
with the following modifications:
(1) No change of Sa for structural periods < 0.5 sec
(2) 20% increase of Sa for structural periods $\geq$ 1 sec
(3) Linear interpolation for structural periods between 0.5 and 1 sec

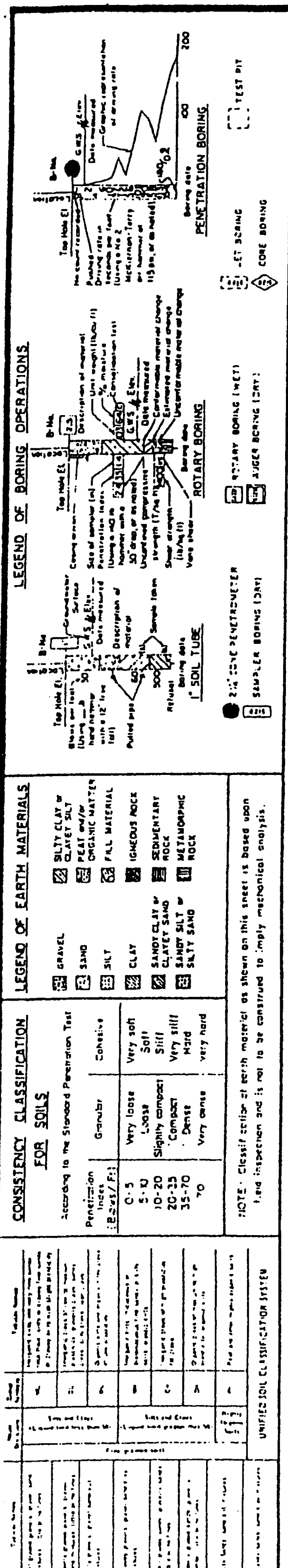
3. Recommended Design Curve = Envelope of above two curves

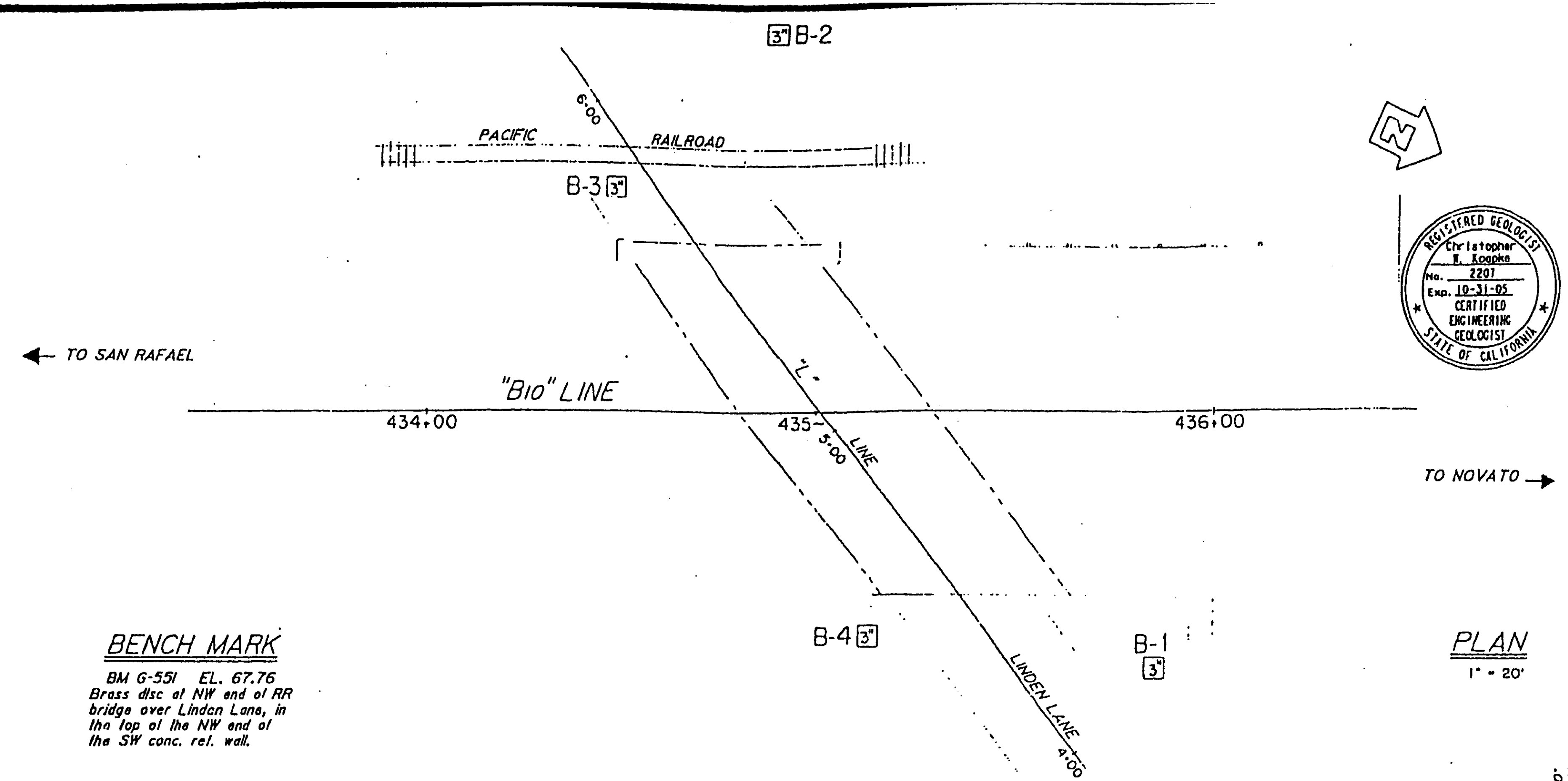
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

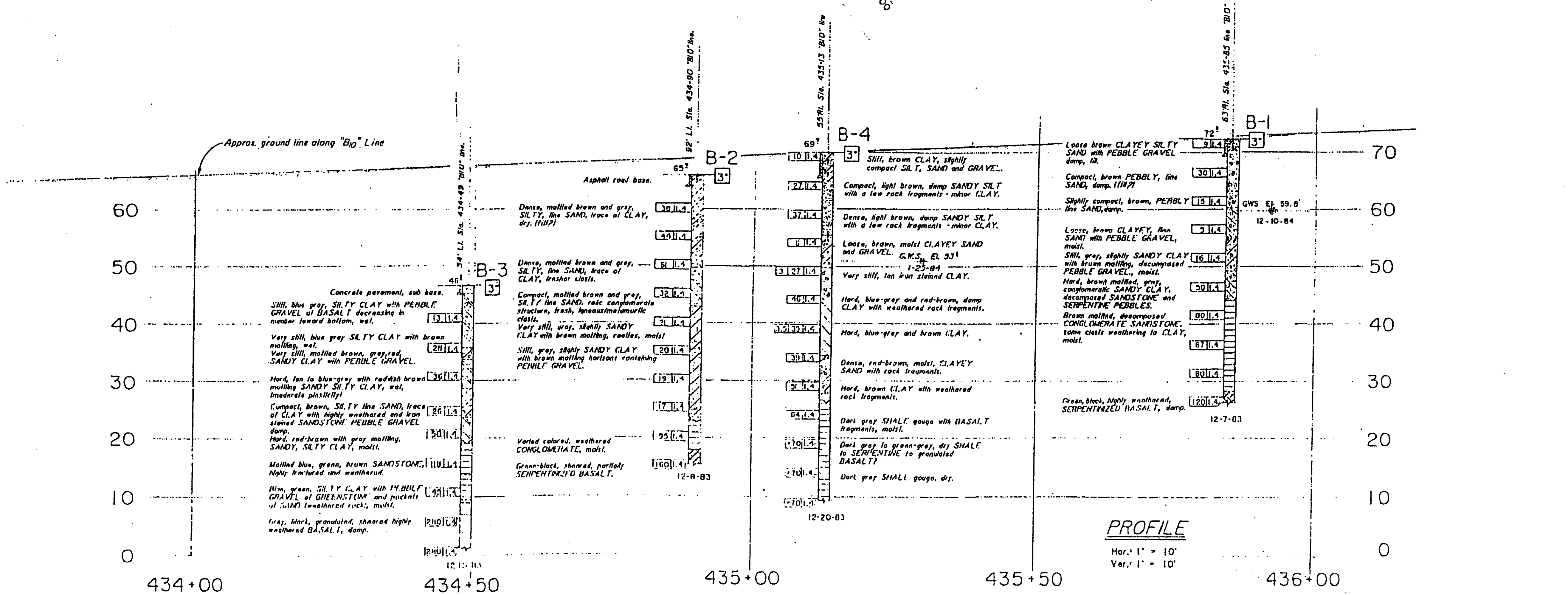
LINDEN LANE BRIDGE
SAN RAFAEL, CALIFORNIA

JOB NO.: 205152.10

PLATE NO.: 5







04	MRN	101

ENGINEERING GEOLOGIST — CERTIFIED ENGINEERING GEOLOGIST NUMBER

DATE APPROVED

DIVISION OF ENGINEERING SERVICES - GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
04	Mrn	101			

REGISTERED GEOLOGIST Christopher N. Koopke No. 2201 Exp. 10-31-05 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST — DATE

LINDEN LANE UC (WIDEN)

LOG OF TEST BORINGS

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 04
EA: 226141

BRIDGE No. 27-0034

Sheet 15 of 15

Revisions made to this Log of Test Borings from the original 1983 Log of Test Borings are the addition of the following table and notes:

Boring	Station	Offset from "RE" Line
B-1	14+34	58.5 m Rt
B-2	14+3.5	9.0 m Rt
B-3	13+92	21.0 m Rt
B-4	14+13	56.0 m Rt

Notes:

1. See the General Plan and/or Foundation Plan for Metric Stationing.
2. Structure Design produced the data presented in the table above. The data are the metric locations for the As-Built Test Borings referenced to the proposed new structure location. This table is presented on the As-Built Log of Test Boring sheet for the convenience of any bidder, contractor or other interested party.

ENGINEERING GEOLOGY AND TECHNICAL SERVICES BRANCH - TRANSPORTATION LABORATORY

DRAWN BY IRMA HOWELL 3/84

CHECKED BY

State of CALIFORNIA
DEPARTMENT OF TRANSPORTATION

STRUCTURES - DESIGN

BRIDGE NO. 27-34
POST MILE 11.6

LINDEN LANE UNDERCROSSING (WIDEN)

LOG OF TEST BORINGS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS

CU 04225
WO 101911

SHEET 17 OF 17

DIST.	COUNTY	ROUTE	KILOMETER POSTS TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Mrn	101	17.5/20.6		

11-30-05
REGISTERED ENGINEER - CIVIL

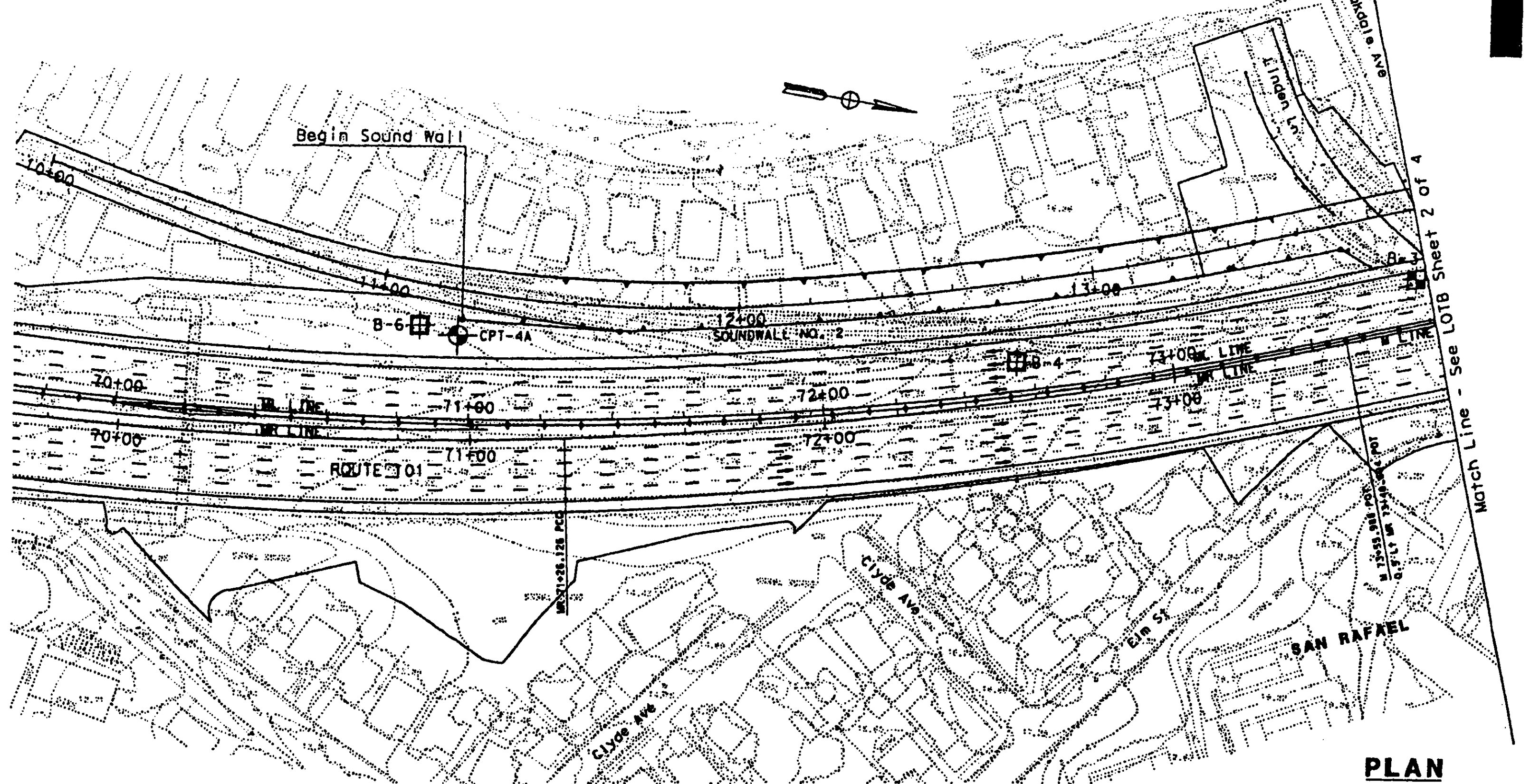
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
P.N. Sundaram
No. 57665
Exp. 12-31-07
CIVIL
STATE OF CALIFORNIA

Cal-Trans now has a web site! To get to the web site, go to: http://tresc.dot.ca.gov

Caltrans Metric

All Dimensions are in Meters unless otherwise shown



PLAN

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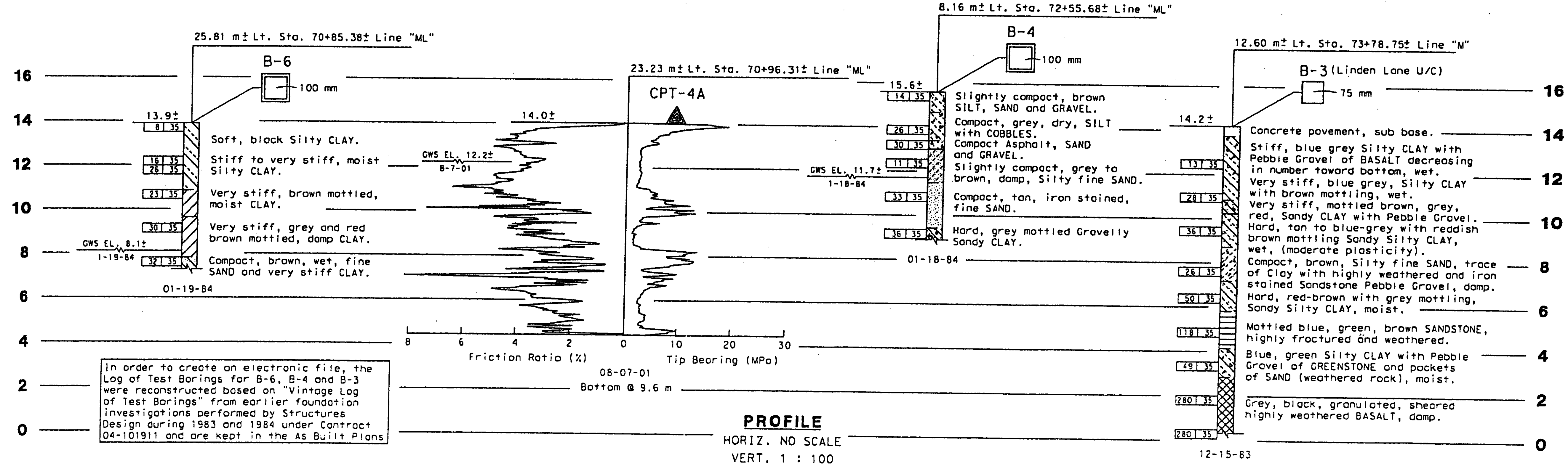
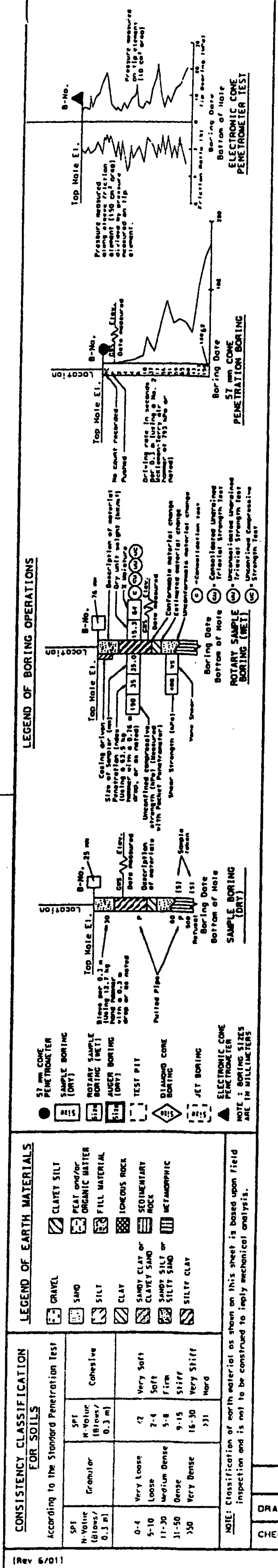


In order to create an electronic file, the Log of Test Borings for B-6, B-4 and B-3 were reconstructed based on "Vintage Log of Test Borings" from earlier foundation investigations performed by Structures Design during 1983 and 1984 under Contract 04-101911 and are kept in the As Built Plans

PROFILE

HORIZ. NO SCALE
VERT. 1 : 100



SW-36

DIVISION OF ENGINEERING SERVICES

DRAWN BY	M. Reynolds; M. Mehta 09/01	
CHECKED BY	P. Sundaram	FIELD INVESTIGATION BY: P. Sundaram (CPT)

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

GEOTECHNICAL SERVICES
OFFICE OF GEOTECHNICAL DESIGN - WEST

BRIDGE NO.
KILOMETER POST

SOUND WALL NO. 2
LOG OF TEST BORINGS

CU 04267
EA 226141

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 1 OF 4

(Rev 6/01)

USERNAME => cleung
DGN FILE => 422614rb032.dgn

DATE PLOTTED => 07-FEB-2006
TIME PLOTTED => 12:28

DIST.	COUNTY	ROUTE	KILOMETER POSTS TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Mrn	101	17.5/20.6		

Caltrans Metric

11-30-05

REGISTERED ENGINEER - CIVIL

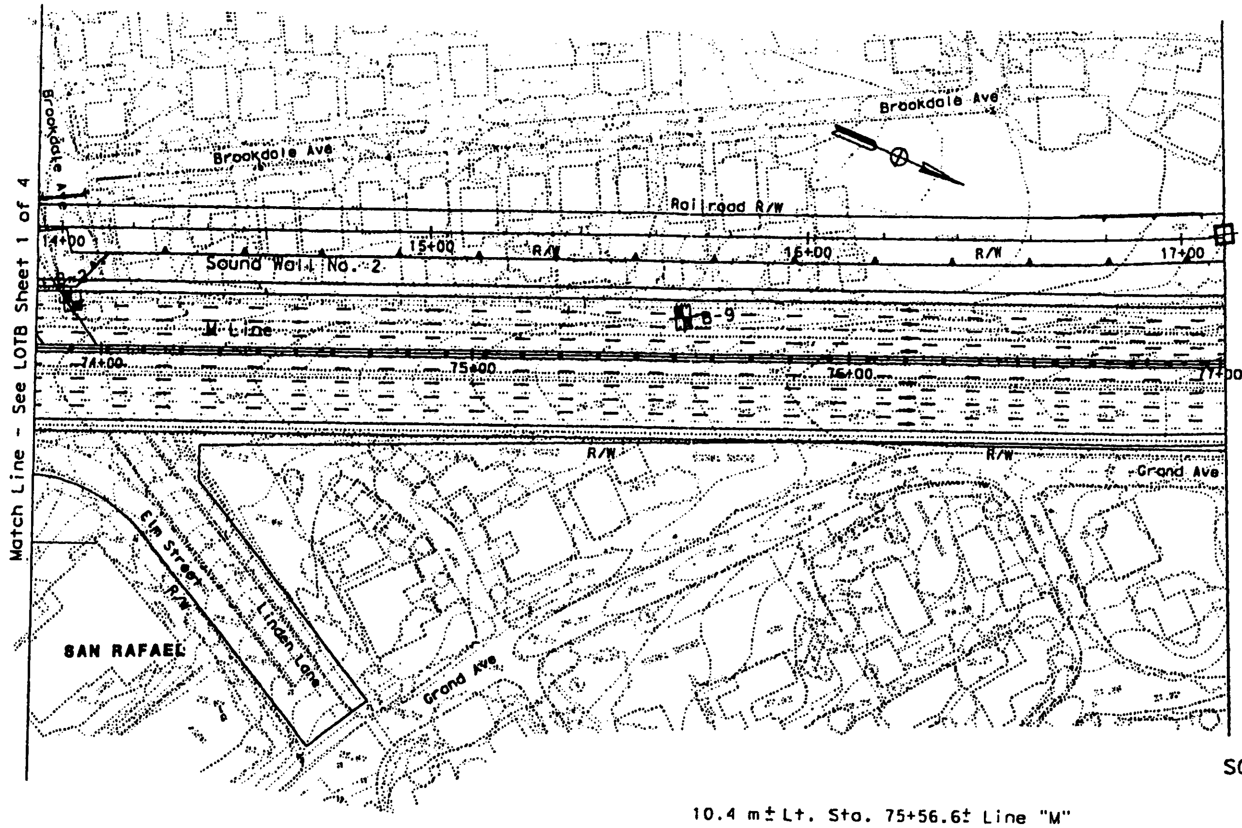
REGISTERED PROFESSIONAL ENGINEER P.N. Sundaram No. 57665 Exp. 2-31-07 CIVIL STATE OF CALIFORNIA

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

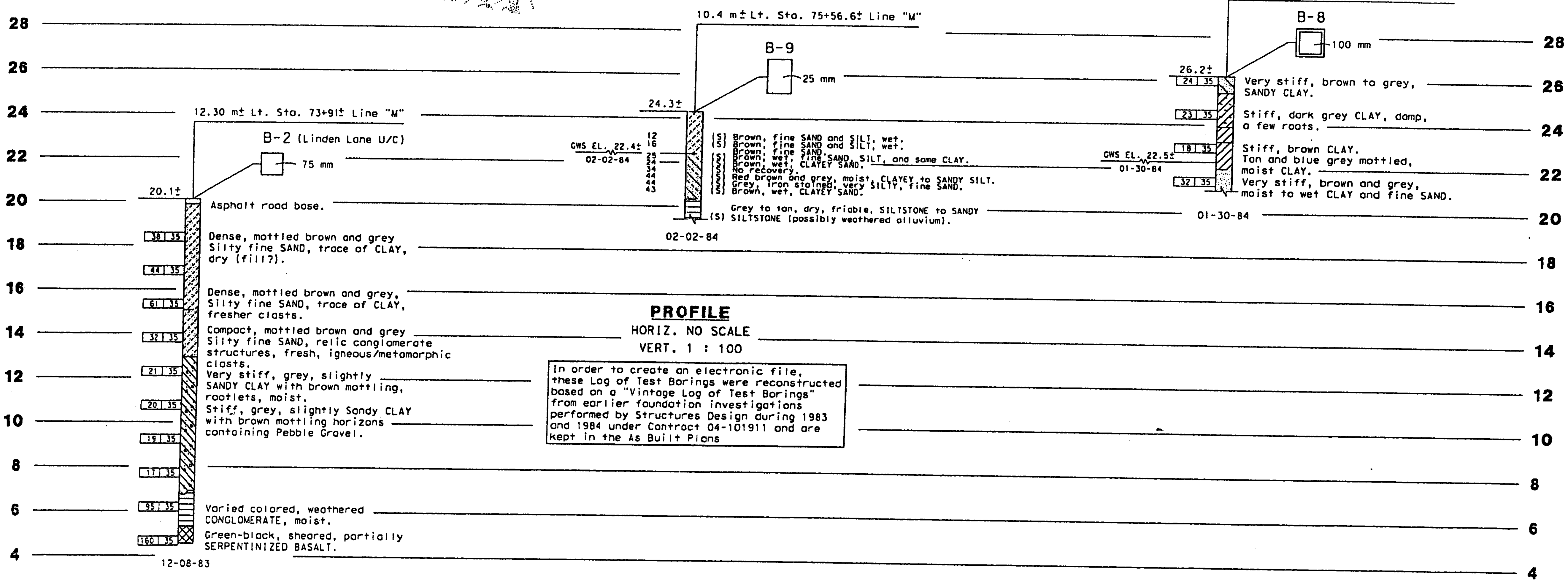
Cal-Trans now has a web site! To get to the web site, go to: http://tresc.dot.ca.gov

All Dimensions are in Meters unless otherwise shown



PLAN

SCALE 1 : 1000

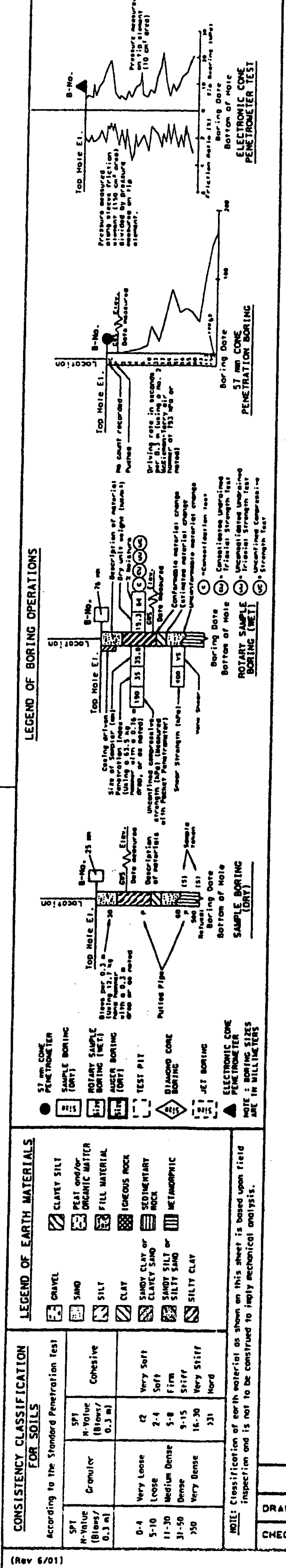


PROFILE

HORIZ. NO SCALE

VERT. 1 : 100

In order to create an electronic file, these Log of Test Borings were reconstructed based on a "Vintage Log of Test Borings" from earlier foundation investigations performed by Structures Design during 1983 and 1984 under Contract 04-101911 and are kept in the As Built Plans



SW-37

DIVISION OF ENGINEERING SERVICES

DRAWN BY	M. Reynolds; M. Mehta 09/01
CHECKED BY	P. Sundaram

FIELD INVESTIGATION BY:

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

GEOTECHNICAL SERVICES OFFICE OF GEOTECHNICAL DESIGN - WEST

BRIDGE NO.

KILOMETER POST

SOUND WALL NO. 2
LOG OF TEST BORINGS

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04267
EA 226141

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 2 OF 4

USERNAME => cleung
DGN FILE => 422614rb033.dgn

DATE PLOTTED => 07-FEB-2006
TIME PLOTTED => 12:28

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 03/24/2006
Date Submitted 03/20/2006

To: Prav Dayah
Parikh Consultants, Inc.
356 S. Milpitas Blvd.
Milpitas, Ca 95035

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 205152.LCN\PUERTO SU Site ID : B-2 #2@11'.
Thank you for your business.

* For future reference to this analysis please use SUN # 47122-93402.

EVALUATION FOR SOIL CORROSION

Soil pH	6.81		
Minimum Resistivity	2.57	ohm-cm (x1000)	
Chloride	5.7 ppm	00.00057	%
Sulfate	61.0 ppm	00.00610	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

PLATE NO.: B-1

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 03/24/2006
Date Submitted 03/20/2006

To: Prav Dayah
Parikh Consultants, Inc.
356 S. Milpitas Blvd.
Milpitas, Ca 95035

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 205152.LCN\PUERTO SU Site ID : B-5 #2@6'.
Thank you for your business.

* For future reference to this analysis please use SUN # 47122-93401.

EVALUATION FOR SOIL CORROSION

Soil pH	6.94		
Minimum Resistivity	2.60	ohm-cm (x1000)	
Chloride	8.9 ppm	00.00089	%
Sulfate	100.6 ppm	00.01006	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

PLATE NO.: B-2

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME	***LINDEN LANE BRIDGE***
PROJECT NO.	***205152.10***
BORING NO.	***B-3 (SOUTH ABUT)***
BOREHOLE DIA (in)=	3
GW DEPTH (ft)=	15

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SIL
2. CLAYS AND PLASTIC SILTS

HAMMER TYPE (1/2) = 1
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
SAN ANDREAS FAULT

a_{max} *(g)=*	0.4
FAULT M_w =	8
MSF =	0.94

Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	*CYCLIC STRESS RATIO (CSR)* σ_v (psf)	σ_v' (psf)	γ_d	CSR	*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)* SPT-N_{60}	$(N_1)_{60}$	F.C.	$(N_1)_{60,\,cs}$	$CRR_{7.5}$	*F.S.=($CRR_{7.5}$/CSR)*MSF*$K\sigma$*$K\alpha$* $K\sigma$	$K\alpha$	F.S.
1	5	2	13	SPT	625	625	0.98		13	20.93				1.00	1	
2	10	2	28	SPT	1250	1250	0.96		28	34.00				1.00	1	
3	15	2	36	SPT	1875	1875	0.93		36	37.92				1.00	1	
4	20	1	26	SPT	2500	2188	0.91	0.27	26	28.34		28.34	0.38	0.97	1	1.30 <- LIQ!!
5	25	2	50	SPT	3125	2500	0.88		50	50.98				0.94	1	
6	30	2	118	SPT	3750	2813	0.84		118	119.41				0.90	1	
7	35	2	49	SPT	4375	3125	0.80		49	47.04				0.87	1	
8	40	2	280	SPT	5000	3438	0.75		280	256.29				0.85	1	
9	45	2	280	SPT	5625	3750	0.70		280	245.38				0.83	1	

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

LINDEN LANE BRIDGE (ABUT 1, FINAL)
205152.10

ALLOWABLE PILE CAPACITY

PILE DIA, B (ft) = 2

SIDE MOBILIZATION
FACTOR (CLAY) = 0.9
FACTOR (SAND) = 0.9

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 2

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO		SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ALLOWABLE COMPRESSION (PER SECTION) (Ton)	ALLOWABLE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
		FTG															
0	5		2	CLAY	0	0	0.0	13	STIFF	0.55	1625			0.00	0.00	7.96	7.96
5	10		2	CLAY	65	325	162.5	25	V.STIFF	0.54	3125			0.00	0.00	15.13	23.09
10	15		2	CLAY	65	325	487.5	25	V.STIFF	0.54	3125			0.00	0.00	15.13	38.23
15	20		1	SAND	65	325	812.5	26	V. DENSE			39	0.94	0.00	0.00	5.37	43.60
20	25		2	CLAY	65	325	1137.5	50	STIFF	0.45	6250			17.89	17.89	25.05	68.65
25	30		2	CLAY	65	325	1462.5	50	STIFF	0.45	6250			17.89	35.78	25.05	93.70
30	35		2	CLAY	65	325	1787.5	50	STIFF	0.45	6250			17.89	53.68	25.05	118.75
35	40		3	IGM	65	325	2112.5	70	IGM (N>50)			52		36.14	89.82	36.14	154.89
40	45		3	IGM	65	325	2437.5	70	IGM (N>50)			51		36.84	126.66	36.84	191.73
45	50		3	IGM	65	325	2762.5	70	IGM (N>50)			51		37.49	164.15	37.49	229.22
50	55		3	IGM	65	325	3087.5	70	IGM (N>50)			50		38.10	202.25	38.10	267.32
55	60		3	IGM	65	325	3412.5	70	IGM (N>50)			49		38.67	240.92	38.67	305.99
60	65		3	IGM	65	325	3737.5	70	IGM (N>50)			49		39.22	280.14	39.22	345.21
65	70		3	IGM	65	325	4062.5	70	IGM (N>50)			48		39.75	319.89	39.75	384.96
70	75		3	IGM	65	325	4387.5	70	IGM (N>50)			47		40.25	360.15	40.25	425.22
75	80		3	IGM	65	325	4712.5	70	IGM (N>50)			47		40.75	400.89	40.75	465.96

Ref. Boring: B-3 (Caltrans 12/1983) for Linden Lane UC
Neglect capacity in the upper zone of pile due to a proposed storm drain pipe below the footing. Also neglect capacity above the liquefiable sand layer.

Use 40 ft (12.2 m) pile length for a service capacity of 58 Tons (516 kN). Pile Tip Elev. = 14.25m (cut-off) - 12.2m = 2.05m

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

LINDEN LANE BRIDGE (ABUT 2)
205152.10

PILE DIA, B (ft) = 2

SIDE MOBILIZATION
FACTOR (CLAY) = 0.9
FACTOR (SAND) = 0.9

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 2

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	FTG	SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ALLOWABLE COMPRESSION (PER SECTION) (Ton)	ALLOWABLE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0	5		1	SAND	65	325	162.5	61	V. DENSE			46	1.20	1.38	1.38	1.38	1.38
5	10		1	SAND	65	325	487.5	32	V. DENSE			41	1.13	3.89	5.27	3.89	5.27
10	15		2	CLAY	65	325	812.5	21	V.STIFF	0.55	2625			9.18	14.46	12.86	18.13
15	20		2	CLAY	65	325	1137.5	20	V.STIFF	0.55	2500			8.75	23.21	12.25	30.38
20	25		2	CLAY	65	325	1462.5	19	V.STIFF	0.55	2375			8.31	31.52	11.63	42.01
25	30		2	CLAY	65	325	1787.5	17	V.STIFF	0.55	2125			7.44	38.95	10.41	52.42
30	35		3	IGM	65	325	2112.5	70	IGM (N>50)			52		36.14	75.09	36.14	88.56
35	40		3	IGM	65	325	2437.5	70	IGM (N>50)			51		36.84	111.94	36.84	125.41
40	45		3	IGM	65	325	2762.5	70	IGM (N>50)			51		37.49	149.43	37.49	162.90
45	50		3	IGM	65	325	3087.5	70	IGM (N>50)			50		38.10	187.53	38.10	201.00
50	55		3	IGM	65	325	3412.5	70	IGM (N>50)			49		38.67	226.20	38.67	239.67
55	60		3	IGM	65	325	3737.5	70	IGM (N>50)			49		39.22	265.42	39.22	278.89
60	65		3	IGM	65	325	4062.5	70	IGM (N>50)			48		39.75	305.16	39.75	318.63
65	70		3	IGM	65	325	4387.5	70	IGM (N>50)			47		40.25	345.42	40.25	358.89
70	75		3	IGM	65	325	4712.5	70	IGM (N>50)			47		40.75	386.17	40.75	399.64
75	80		3	IGM	65	325	5037.5	70	IGM (N>50)			46		41.23	427.39	41.23	440.87

Ref. Boring: B-2 (Caltrans 12/1983) for Linden Lane UC
Use 35 ft (10.75m) pile length for a service capacity of 58 Tons (516 kN). Pile Tip Elev. = 16.17m (cut-off) - 10.75m = 5.4m.

```
LPILEP5
LINDEN LANE BRIDGE, ABUT 1, 24" DIA. CIDH (FIXED HEAD)
1       1       0         0        0        0
70      2       60        420      0
0       24      16286.02           452.39   3000000
420     24      16286.02           452.39   3000000
3       6       6         0        2
3       60      180       0        0
10      180     240       0        0
3       240     420       0        0
60      0.038
180     0.038
180     0.038
240     0.038
240     0.038
420     0.038
60      17.36   0         0.005    0
180     17.39   0         0.005    0
180     0       0         0        0
240     0       0         0        0
240     27.78   0         0.004    0
420     27.78   0         0.004    0
60      0.6     1
420     0.6     1
0       1       1
5
2       30000   0         200000
2       35000   0         200000
2       60000   0         200000
2       70000   0         200000
2       80000   0         200000
0
1       1       0
100     1E-5    100
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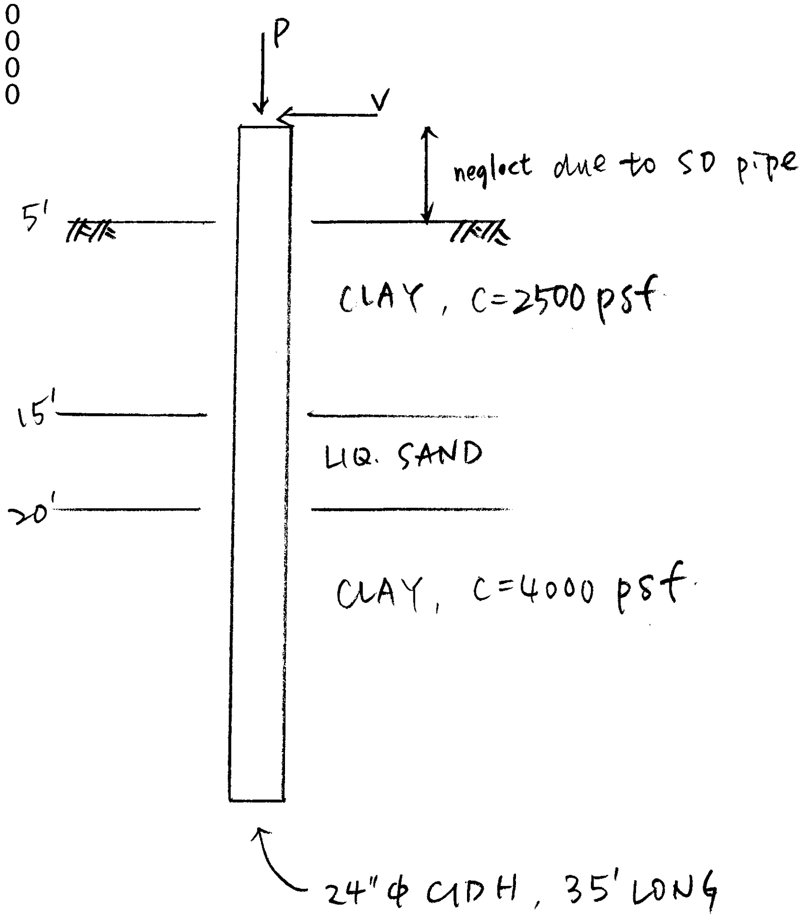
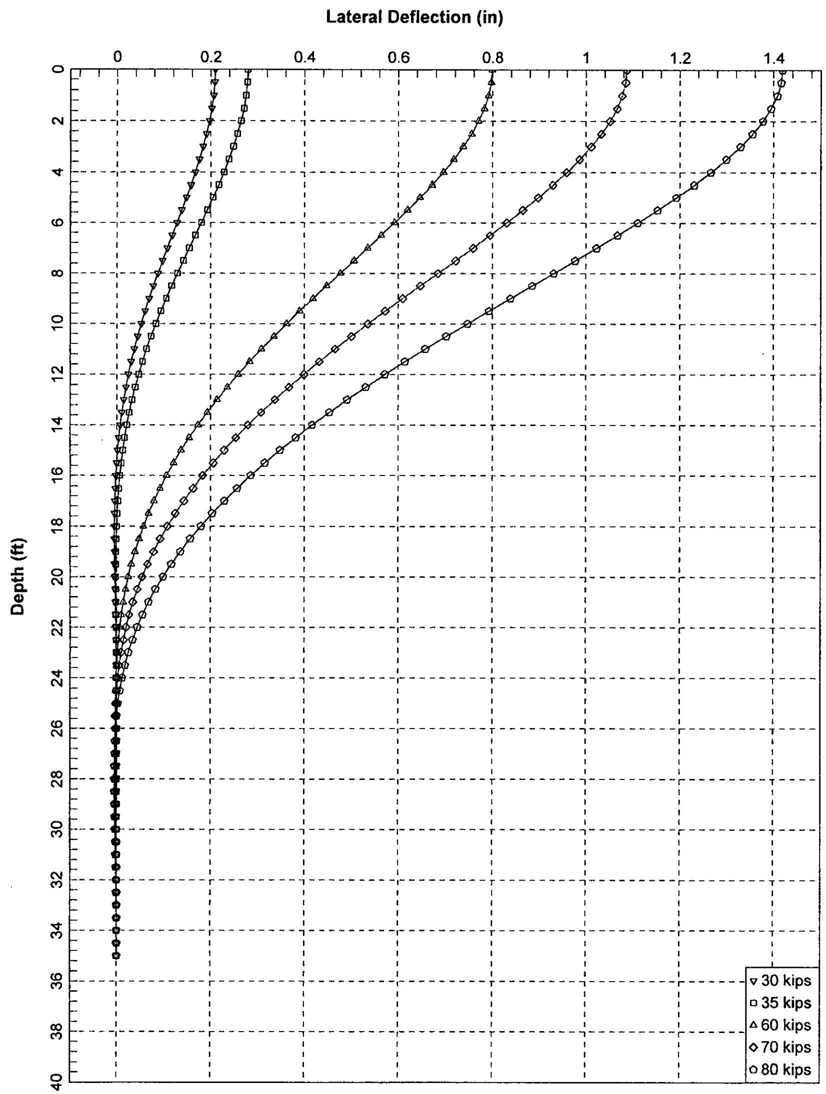
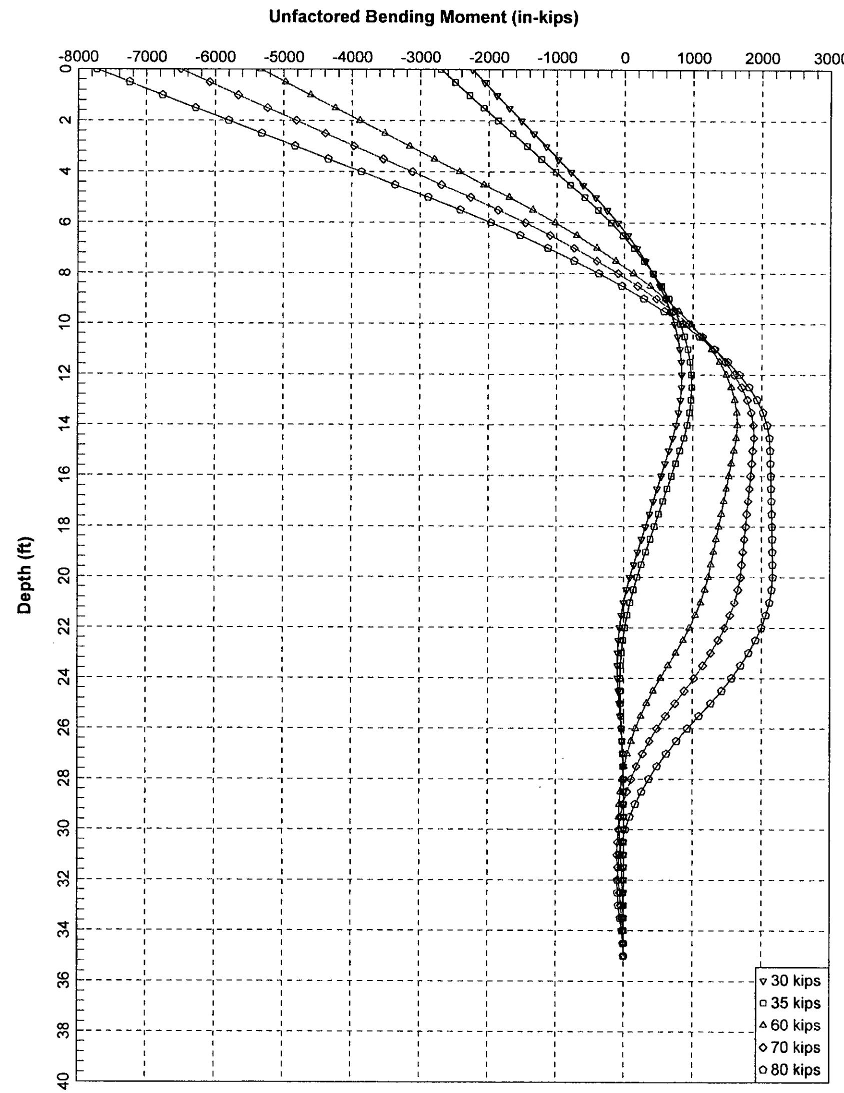
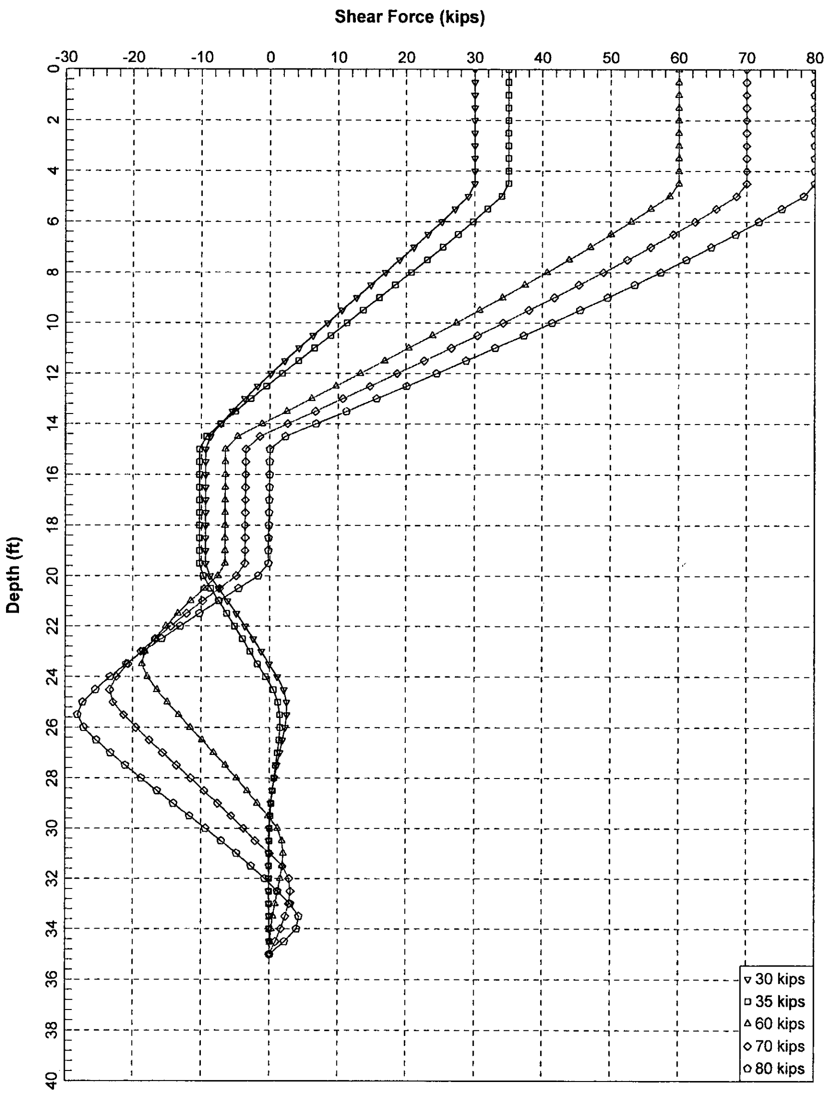




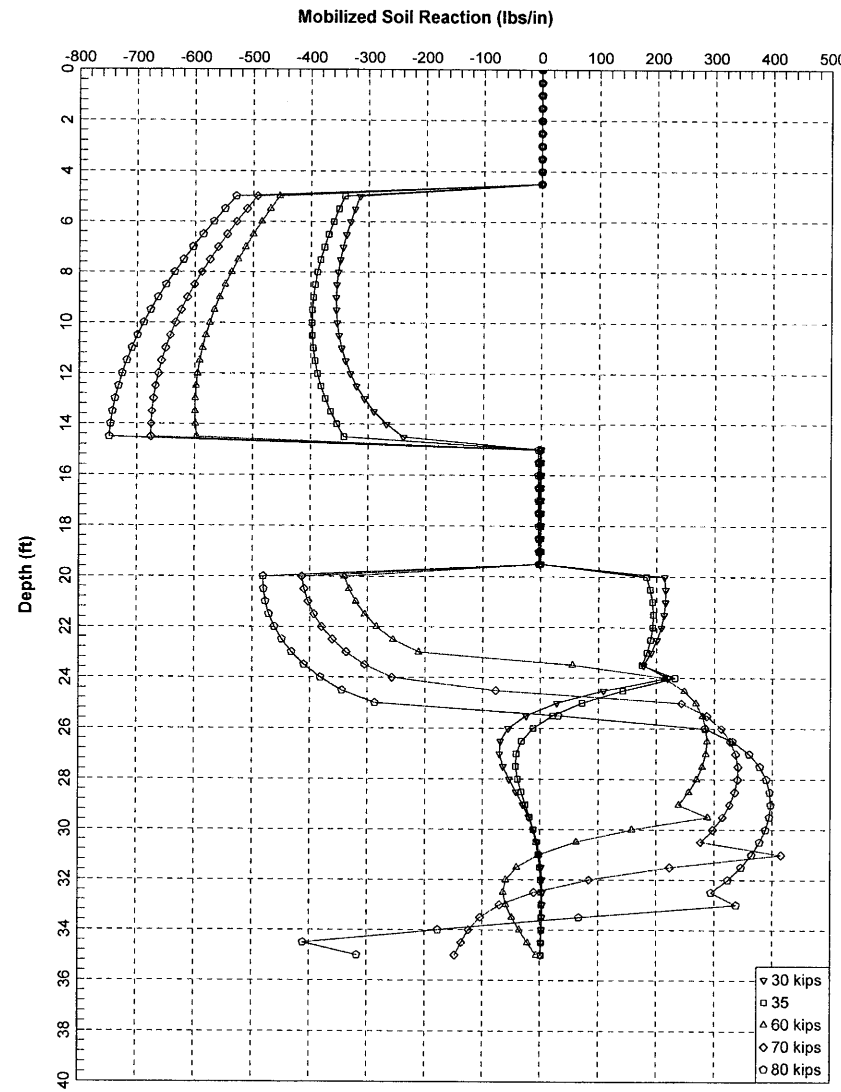
LINDEN LANE BRIDGE, ABUT 1, 24" CIDH (FIXED HEAD, W/ LIQ. SAND)



LINDEN LANE BRIDGE, ABUT 1, 24" CIDH (FIXED HEAD, W/ LIQ. SAND)



LINDEN LANE BRIDGE, ABUT 1, 24" CIDH (FIXED HEAD, W/ LIQ. SAND)



LINDEN LANE BRIDGE, ABUT 1, 24" CIDH (FIXED HEAD, W/ LIQ. SAND)

NOLTE ASSOCIATES
1731 North First Street
San Jose, CA

July 26, 2006
Job No.: 205152.LDN

Attn.: Mr. Michael McNeeley

Sub: Response to Caltrans Comments for Linden Lane Bridge

Ref: Foundation Review Comments
Caltrans Division of Engineering Services
Geotechnical Services
File # 04-MRN-101 PM 10.9/12.8 EA 04-226141

Gentlemen:

Based on the review comments from Caltrans dated May 12, 2006 we have the following responses:

1. Comment: The report refers to borings drilled for adjacent Linden Avenue UC. Please justify why no borings were drilled at the abutment locations.

   Response: The proposed program was to use existing as-built LOTB as much as possible and supplement with additional data, as necessary. Based on the information provided to us, two borings are available from Caltrans files in the close proximity of the proposed abutments of the POC structure. In addition the access is very limited to the proposed bridge abutments other than for a hand held portable rig. Reviewing the LOTB we feel that the portable rig could not have drilled beyond say 10' rendering that effort unsuccessful. Finally, there is always some risk involved in not having a boring at the specific abutment location, however, this structure is planned to be supported on CIDH piles and a conservative approach has been taken for the design and construction of the piles to compensate for such situation. The Project Manager has been made aware of this as well.

2. Comment: Include LOTB sheet for the proposed bridge in the plans.

   Response: Will comply.

Based on the review comments from Caltrans dated June 29, 2006 we have the following responses:

1. Comment: The boring location and numbering are incorrect. Please rectify.
   Response: Will comply.

2. Comment: The groundwater elevation of 16 m assumed in the design is suspect and is probably too high. Depending on the groundwater elevation, the medium dense sand layer may liquefy. The liquefaction analysis in the report is based on assumptions that are highly suspect.
   Response: The groundwater has been interpolated based on the groundwater elevations in adjacent borings. This is generally within the layers that have been observed in other borings. From a design and construction standpoint the CIDH pile should assume that groundwater be at the level anticipated.
   Based on our analysis, the medium dense sand layer will be subject to liquefaction when the groundwater level is lower than Elev. 10m, and the post-liquefaction settlement is anticipated to be on the order of 13 mm (0.5"). Down drag force should be minimal for such small movement. Therefore, we have neglected the vertical capacity in the upper zone of the pile. Based on the boring data, competent material was encountered below the sand layer. The pile tip elevation of 3.5 m is still valid for the design service load of 450 kN. Lateral capacity is not affected. The calculation will be attached in the final report.

3. Comment: The corrosion data used for corrosion recommendation is from boring about 2950 ft away. The recommendations do not seem reasonable.
   Response: Please see item #1 response. It is correct that the data is used from a distant boring. However it is also based on similar soil conditions.

4. Comment: Lack of borings at the site makes the design suspect and increase the possibility of differing site condition claims. We are not confident that the old borings are sufficient for design at this location.
   Response: Please see item #1 response. It is not feasible to do conventional borings without undertaking a major construction effort and physically creating access roads. This was deemed to be not a reasonable approach considering that the Agency is willing to take some risks with differing site conditions.

5. Comment: The pile tip is controlled by axial capacity: yet there is no discussion on the axial capacity and no back up calculations are included in the Appendix. Please rectify.
   Response: Will comply.

6. Comment: Include a LOTB specific to the bridge structure that will be included in the project plans.
   Response: The as-built LOTB that includes the two borings B-2 and B-3 by Caltrans will be included.

Very truly yours,
PARIKH CONSULTANTS, INC.

Frank Y. Wang, P.E., 67751
Project Engineer

Gary Parikh, P.E., G.E., 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-07
GEOTECHNICAL
STATE OF CALIFORNIA

Attachment: Caltrans Review Comments
Backup Calculations

FW {S:\Ongoing Projects\205152.10\Comments & Responses\}

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

TO: MR. EARL SEABERG, CHIEF
Office of Special Funded Projects (OSFP)
Attention: Tracy Bertram

DATE: 5/12/06

FILE: 04 (District) | MRN (County) | 101 (Route) | 17.5/20.6 (KP)

FDN REPORT BY: Parikh Consultants, Inc. DATED: 4/06

Structure Name: Linden Lane Bridge

GENERAL PLAN DATED: 4/3/06 FDN PLAN DATED: No Date

EA Number: 04-TLB27t Bridge Number: ?

Submittal (*Check One*): [x] 1st [ ] 2nd [ ] 3rd [ ] 4th [ ] Other: ______

1. The report refers to borings drilled for adjacent Linden Avenue UC. Please justify why no borings were drilled at the abutment locations.
2. Include LOTB sheet for the proposed bridge in the plans.

Registered Professional Engineer
Wajahat Nyaz
No. GE2577
Exp: 06/30/07
GeoTechnical
State of California

[ ] Local Assistance Project [x] Special Funded Project

Approval: C3 - Not Approved

Office of Special Funded Projects (OSFP)

Wajahat Nyaz
Office of Geotechnical Design - West

cc: GS (Oakland, 2 copies), Lab File Room (Sacramento) DES Specifications and Estimates (All Reviews) Structure Construction R.E. Pending File

Revised 09/03

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

RB
06-29-06

TO: MR. EARL SEABERG, CHIEF
Office of Special Funded Projects (OSFP)
Attention: Tracy Bertram

DATE: ~~04/18/06~~

FILE: 04 (District) MRN (County) 101 (Route) 10.9/12.8 (PM)

FDN REPORT BY: Parikh Inc. DATED: 05/2006 Linden Lane Bridge (Structure Name)

GENERAL PLAN DATED: 4/3/06 FDN PLAN DATED: NA 04-226141 (EA Number) 270117Z (Bridge Number)

Submittal (*Check One*): [ ] 1st [x] 2nd [ ] 3rd [ ] 4th [ ] Other: ______

Please note that the FR for this bridge was reviewed by our office under EA 04-TLB27t on 5/12/06. Our comments have not been addressed. We also have some additional comments that expand on our earlier comments

1. The boring location and numbering are incorrect. Please rectify.
2. The groundwater elevation of 16 m assumed in the design is suspect and is probably too high. Depending on the groundwater elevation, the medium dense sand layer may liquefy. The liquefaction analysis in the report is based on assumptions that are highly suspect.
3. The corrosion data used for corrosion recommendation is from boring about 2950 ft away. The recommendations do not seem reasonable.
4. Lack of borings at the site makes the design suspect and increase the possibility of differing site condition claims. We are not confident that the old borings are sufficient for design at this location.
5. The pile tip is controlled by axial capacity; yet there is no discussion on the axial capacity and no back up calculations are included in the Appendix. Please rectify.
6. Include a LOTB specific to the bridge structure that will be included in the project plans.

Registered Professional Engineer
Wajahat Nyaz
No. GE2577
Exp: 06/30/07
GeoTechnical
State of California

[ ] Local Assistance Project [x] Special Funded Project

Approval: C3 - Not Approved

Office of Special Funded Projects (OSFP)

Tung Nguyen/Wajahat Nyaz
Office of Geotechnical Design - West

cc: GS (Oakland, 2 copies), Lab File Room (Sacramento) DES Specifications and Estimates (All Reviews) Structure Construction R.E. Pending File

Revised 09/03

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME	***LINDEN LANE BRIDGE***	*SOIL GROUPS*	*FAULT INFO*
PROJECT NO.	***205152.10***	1. GRAVELS, SANDS AND NONPLASTIC SIL⁻	*SAN ANDREAS FAULT*
BORING NO.	***B-3 (SOUTH ABUT)***	2. CLAYS AND PLASTIC SILTS	*a_{max} (g)=* 0.4
			FAULT M_w = 8
BOREHOLE DIA (in)=	3	*HAMMER TYPE (1/2) =* 1	
GW DEPTH (ft)=	15	(1. ROPE AND PULLEY; 2. AUTOMATIC)	*MSF =* 0.94

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*					*F.S.=($CRR_{7.5}$/CSR)*MSF*$K\sigma$*$K\alpha$*		
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	$(N_1)_{60}$	F.C.	$(N_1)_{60, CS}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
1	5	2	13	SPT	625	625	0.98		13	20.93				1.00	1	
2	10	2	28	SPT	1250	1250	0.96		28	34.00				1.00	1	
3	15	2	36	SPT	1875	1875	0.93		36	37.92				1.00	1	
4	20	1	26	SPT	2500	2188	0.91	0.27	26	28.34		28.34	0.38	0.97	1	1.30 <- LIQ!!
5	25	2	50	SPT	3125	2500	0.88		50	50.98				0.94	1	
6	30	2	118	SPT	3750	2813	0.84		118	119.41				0.90	1	
7	35	2	49	SPT	4375	3125	0.80		49	47.04				0.87	1	
8	40	2	280	SPT	5000	3438	0.75		280	256.29				0.85	1	
9	45	2	280	SPT	5625	3750	0.70		280	245.38				0.83	1	

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

LINDEN LANE BRIDGE (ABUT 1, FINAL)
205152.10

ALLOWABLE PILE CAPACITY

PILE DIA, B (ft) = 2

SIDE MOBILIZATION
FACTOR (CLAY) = 0.9
FACTOR (SAND) = 0.9

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 2

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO		SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ALLOWABLE COMPRESSION (PER SECTION) (Ton)	ALLOWABLE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
		FTG															
0	5		2	CLAY	0	0	0.0	13	STIFF	0.55	1625			0.00	0.00	7.96	7.96
5	10		2	CLAY	65	325	162.5	25	V.STIFF	0.54	3125			0.00	0.00	15.13	23.09
10	15		2	CLAY	65	325	487.5	25	V.STIFF	0.54	3125			0.00	0.00	15.13	38.23
15	20		1	SAND	65	325	812.5	26	V. DENSE			39	0.94	0.00	0.00	5.37	43.60
20	25		2	CLAY	65	325	1137.5	50	STIFF	0.45	6250			17.89	17.89	25.05	68.65
25	30		2	CLAY	65	325	1462.5	50	STIFF	0.45	6250			17.89	35.78	25.05	93.70
30	35		2	CLAY	65	325	1787.5	50	STIFF	0.45	6250			17.89	53.68	25.05	118.75
35	40		3	IGM	65	325	2112.5	70	IGM (N>50)			52		36.14	89.82	36.14	154.89
40	45		3	IGM	65	325	2437.5	70	IGM (N>50)			51		36.84	126.66	36.84	191.73
45	50		3	IGM	65	325	2762.5	70	IGM (N>50)			51		37.49	164.15	37.49	229.22
50	55		3	IGM	65	325	3087.5	70	IGM (N>50)			50		38.10	202.25	38.10	267.32
55	60		3	IGM	65	325	3412.5	70	IGM (N>50)			49		38.67	240.92	38.67	305.99
60	65		3	IGM	65	325	3737.5	70	IGM (N>50)			49		39.22	280.14	39.22	345.21
65	70		3	IGM	65	325	4062.5	70	IGM (N>50)			48		39.75	319.89	39.75	384.96
70	75		3	IGM	65	325	4387.5	70	IGM (N>50)			47		40.25	360.15	40.25	425.22
75	80		3	IGM	65	325	4712.5	70	IGM (N>50)			47		40.75	400.89	40.75	465.96

Ref. Boring: B-3 (Caltrans 12/1983) for Linden Lane UC
Neglect capacity in the upper zone of pile due to a proposed storm drain pipe below the footing. Also neglect capacity above the liquefiable sand layer.

Use 40 ft (12.2 m) pile length for a service capacity of 58 Tons (516 kN). Pile Tip Elev. = 14.25m (cut-off) - 12.2m = 2.05m

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

LINDEN LANE BRIDGE (ABUT 2)
205152.10

PILE DIA, B (ft) = 2

SIDE MOBILIZATION
FACTOR (CLAY) = 0.9
FACTOR (SAND) = 0.9

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 2

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	FTG	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ALLOWABLE COMPRESSION (PER SECTION) (Ton)	ALLOWABLE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0	5		1	SAND	65	325	162.5	61	V. DENSE			46	1.20	1.38	1.38	1.38	1.38
5	10		1	SAND	65	325	487.5	32	V. DENSE			41	1.13	3.89	5.27	3.89	5.27
10	15		2	CLAY	65	325	812.5	21	V.STIFF	0.55	2625			9.18	14.46	12.86	18.13
15	20		2	CLAY	65	325	1137.5	20	V.STIFF	0.55	2500			8.75	23.21	12.25	30.38
20	25		2	CLAY	65	325	1462.5	19	V.STIFF	0.55	2375			8.31	31.52	11.63	42.01
25	30		2	CLAY	65	325	1787.5	17	V.STIFF	0.55	2125			7.44	38.95	10.41	52.42
30	35		3	IGM	65	325	2112.5	70	IGM (N>50)			52		36.14	75.09	36.14	88.56
35	40		3	IGM	65	325	2437.5	70	IGM (N>50)			51		36.84	111.94	36.84	125.41
40	45		3	IGM	65	325	2762.5	70	IGM (N>50)			51		37.49	149.43	37.49	162.90
45	50		3	IGM	65	325	3087.5	70	IGM (N>50)			50		38.10	187.53	38.10	201.00
50	55		3	IGM	65	325	3412.5	70	IGM (N>50)			49		38.67	226.20	38.67	239.67
55	60		3	IGM	65	325	3737.5	70	IGM (N>50)			49		39.22	265.42	39.22	278.89
60	65		3	IGM	65	325	4062.5	70	IGM (N>50)			48		39.75	305.16	39.75	318.63
65	70		3	IGM	65	325	4387.5	70	IGM (N>50)			47		40.25	345.42	40.25	358.89
70	75		3	IGM	65	325	4712.5	70	IGM (N>50)			47		40.75	386.17	40.75	399.64
75	80		3	IGM	65	325	5037.5	70	IGM (N>50)			46		41.23	427.39	41.23	440.87

Ref. Boring: B-2 (Caltrans 12/1983) for Linden Lane UC
Use 35 ft (10.75m) pile length for a service capacity of 58 Tons (516 kN). Pile Tip Elev. = 16.17m (cut-off) - 10.75m = 5.4m.